Nonlinear static response of a masonry infilled RC building stock considering the variability of infill parameters

G. Mucedero[(1)], D. Perrone[(2)], R. Monteiro[(3)]

[(1)] *PhD Student, University School for Advanced Studies IUSS Pavia, Pavia, Italy, gianrocco.mucedero@iusspavia.it*
[(2)] *Postdoctoral Researcher, University School for Advanced Studies IUSS Pavia, Pavia, Italy, daniele.perrone@iusspavia.it*
[(3)] *Associate Professor, University School for Advanced Studies IUSS Pavia, Pavia, Italy, ricardo.monteiro@iusspavia.it*

Abstract

The seismic risk assessment of existing masonry infilled reinforced concrete (RC) building portfolios is a highly relevant issue in Italy and other Mediterranean countries, particularly in regions where a large part of the built environment was not designed according to modern seismic codes. In the past, masonry infills were generally considered as non-structural elements hence not accounted for in the design process, whereas the results of experimental tests and past earthquake evidence have demonstrated how they strongly influence the global and local behavior of RC buildings. This paper addresses the characterization of the large-scale seismic performance of different types of masonry infilled existing buildings, with a particular focus on the uncertainty surrounding the definition of the properties and modelling of the masonry infills. To accomplish this goal, the study starts with the development of a fully integrated building stock, based on statistical data, and representative of masonry infilled RC buildings designed according to the Italian codes in force between 1960 and 1980, using a simulated design approach. The main typologies of masonry infill used in Italy, which are affected by significant dispersion, were considered in terms of material properties, thickness, presence of openings and manufacturing techniques. Statistical data and the results of experimental tests carried out on RC infilled frames were used to identify the main masonry infill typologies. Detailed numerical models able to predict the features of the RC frames were defined and assessed through the results of cyclic experimental tests available in the literature. Moreover, different numerical models to define the backbone hysteresis of masonry infills have been adopted to further characterize the uncertainties surrounding the infills. Finally, the impact of the observed uncertainty on the seismic vulnerability of the developed RC building portfolio has been assessed in terms of maximum base shear capacity.

Keywords: masonry infills, existing RC buildings, building portfolios, large-scale assessment

1. Introduction

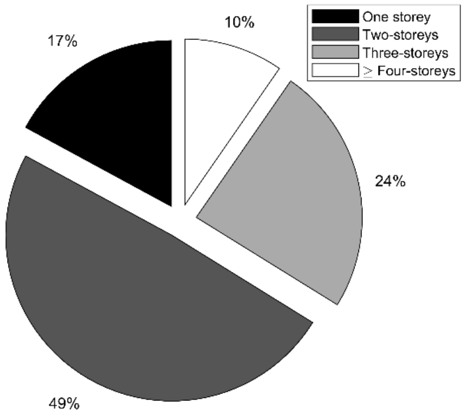
The seismic vulnerability assessment of existing masonry infilled reinforced concrete (RC) buildings is a highly relevant issue in Italy and other Mediterranean countries, particularly in regions where a large part of the built environment was not designed according to modern seismic codes. The results of experimental and numerical studies carried out in the last decade demonstrated the relevant role played by masonry infills in the global and local response of RC buildings, which has also been observed during past earthquakes [1]. The interaction between the masonry infill panels and the surrounding frame, combined with the frequently poor seismic detailing in existing RC buildings, can lead to possible local shear failure of columns, short column effects or soft story mechanism during an earthquake. Based on these considerations, within seismic risk assessment of infilled RC frames, both at single building and regional scale, the definition of material properties and numerical modelling approaches both of RC members and masonry infills is of paramount importance. According to the Italian National Institute of Statistics (ISTAT) [2] about 80% of the Italian building stock (over 12 million buildings) are residential, of which around 30% are in RC. Although this percentage is lower, when compared to the one of unreinforced masonry buildings (URM), which are typically highly vulnerable, the seismic risk and economic losses surrounding the former structural typology can become extremely high due to the higher number of floors, dwellings and occupants in each dwelling, which characterize the RC buildings with respect to URM buildings. Regional and/or national-scale approaches are evermore used for seismic risk assessment of buildings, by governmental decision makers who need to decide and justify the distribution of limited financial resources that aim to reduce the overall seismic risk of the Italian building stock, with special focus on critical facilities. In this sense, a regional scale approach should include and reflect, as much as possible, the geometrical and material variability representative of the area of interest, mostly because the parameters that affect the building-to-building variability characterizing any selected building stock is affected by a number of uncertainties. Currently, there are several large-scale studies focusing on the seismic assessment of RC building portfolios in Mediterranean countries [3]. Despite such abundance, no particular attention has paid to the uncertainty surrounding the mechanical properties of the infills and how this might affect the fragility of the entire building stock. Even in cases where the masonry infills have been considered through simplified approaches, the variabilities related to the geometrical configuration, material properties and modelling assumptions were not taken into account. However, the experimental studies available in the literature, as well as the results of in-situ survey carried out around Italy, pointed out that the masonry infills are affected by significant variability, due to material properties, thickness, presence of openings and manufacturing techniques. Based on these considerations, the assumption of neglecting the masonry infill variability might be reductive and lead to results that are not realistic. This research study is focused on the large-scale characterization of the seismic response of existing masonry infilled residential buildings, through the development of a fully integrated building stock, in which the uncertainty surrounding the definition of the properties and modelling of the masonry infills is accounted for.

2. Italian masonry-infilled RC residential building portfolio

The need for large-scale fragility and risk models characterizing the response of portfolios of buildings, rather than single buildings, has been demonstrated by the large human and economic losses observed following major earthquake that struck densely populated regions. Fragility models at regional scale allow to allocate national funds available for retrofit strategies to those regions where seismic risk is higher. At the same time, the development of a fragility model at regional and/or national scale requires a high level of knowledge about the properties and uncertainties in existing building portfolios.

This study makes use of a fully integrated building stock, put together from scratch, representing existing masonry-infilled RC residential buildings in Italy. This population of buildings accounts for both the geometrical variability of the buildings and the infills variability. To this aim, statistical data collected from ISTAT [2] and from a database comprising information regarding experimental tests carried out on different masonry infill typologies were used [4]. According to the data provided by ISTAT [2], in Italy, the RC residential buildings' class includes 3 594 695 residential buildings, representing around 30% of the entire building stock. These buildings can be disaggregated on the construction age, which allows to identify the building codes employed for their design, the material properties, construction practices and structural detailing. As reported in Fig.1 (left), most of the RC residential buildings in Italy were built before 1990 and, analyzing the ISTAT database, no particular correlation between period of construction and geographical areas has been found, demonstrating a quasi-uniform building construction trend on the national territory. It is worth mentioning that, between 1960 and 1980, there were more new buildings in the Nord-West and South of Italy than in other areas.

(a)



(b)

Fig. 1 (a) Residential buildings by construction age and by (b) number of storeys

Following such a preliminary analysis, this study focused on RC buildings built between 1970 and 1980, one of the decades with massive construction of such building typology and during which an update of the seismic design guidelines was introduced. In order to fully develop the building portfolio, the variability in the following parameters has been studied and taken into account: plan dimension, number of floors, interstory height, span lengths, material properties of RC elements and of masonry infills.

2.1. Building geometry

Different sources of variability were considered in the definition of the building stock. The ISTAT database does not provide information about the buildings' plan dimensions [2] however, for residential buildings, it could be estimated using data regarding urbanistic directives and population distribution. By combining this information with satellite images, on which direct measurements of the buildings in-plan dimensions were carried out [5] or different geographical areas in Italy, a more precise and accurate definition of the plan dimensions was achieved: the range of the longest and shortest dimensions of the building (L_x and L_y) is between 10-50m and 8-25m, respectively. The mean values of L_x and L_y are almost in accordance with those presented in [6], in which data about the parameters affecting the seismic vulnerability of RC buildings in the Benevento and Catania regions (South and Islands) are provided. The dimension ranges presented here are wider, when compared to those presented in [8], due to information and building measurements in different geographical areas and residential building typologies in Italy. The longest dimensions are found in the metropolitan and major cities, where extensive housing estates were built, whereas the smallest ones are generally found in independent residential buildings, for which a plan area smaller than 80m^2 is very likely. As shown in Fig.1 (right), when classifying the residential buildings by number of floors, including the ground floor, almost half of the buildings consist of two floors, whilst only 10% have four or more floors. Furthermore, the data provided by ISTAT [2] aggregated all the buildings with more than 4 floors into a single class, making it impossible to find out the occurrences by number of floors. Based on these considerations, the defined building stock consists of frames from 1 to 6 floors. Considering the design practice and common architectural trends, the interstory height was fixed at 3.0m and span lengths were limited to the range of 3-5m. No information is provided in [2] about structural component properties. In order to reach a more detailed building portfolio, additional information was collected from original drawings of residential buildings built in different geographical areas in Italy.

2.2. Material properties

The mechanical properties of the materials used in the selected construction period (1970-1980) were obtained from the results of in situ tests on existing buildings or from the data collected in laboratory archives [9-10]. As regards the mechanical characteristics of the reinforcement bars used in Italy between 1950 and 1980, very detailed information is provided in [9], based on a large number of archive data from the steel testing laboratory at the University of Naples Federico II. Steel rebars FeB22K and FeB32K were identified as the most used typologies in the period between 1974 and 1980; these rebars are characterized by a mean yielding strength of 325.11MPa and 430.03MPa, respectively. In the period 1970-1974, five different steel typologies, namely COM, AQ42, AQ50, AQ60 and ALE, were used hence, considering the wide range of steel properties adopted in the reference construction period, it was chosen to use a value of 350MPa for the yield strength. The ratio between the ultimate and the yield strength was assumed as 1.4. As regards the diameter of the longitudinal bars, 14mm was chosen according to the mean value provided in [9].
For what concerns the concrete strength, exhaustive information is presented in [10], from a large database of test results on concrete cores extracted from about 300 RC public buildings located in the Basilicata region (Italy), for which the mean strength and dispersion for different construction periods were provided. Test results on concrete cores shown that the mean values of concrete strength in the construction periods 1961–1971 and 1972–1981 is 19.53MPa and 21.03MPa, respectively. Although these tests were based on concrete strength of hospitals and schools, this information is deemed to be suitable for residential buildings and 20MPa was considered as mean concrete strength in the period 1970-1980.

2.3. Masonry infills

As mentioned earlier, despite the high-level of uncertainty surrounding the masonry infill properties, constant mechanical and geometrical properties of the masonry infills are typically assumed in risk assessment studies both at single-building and regional scale. To the authors best knowledge, the results of in-situ tests on masonry infills are not available. The results of experimental studies [12-14], also reported in recent databases dealing with the definition of the mechanical properties of masonry infills [4], are taken into account to define the variability in the masonry infills' characterization, which is related to many parameters, such as the maximum shear strength of the panel, the stiffness of the panel, the relative stiffness between the panel and the surrounding frame, the vertical/horizontal compressive strength of the masonry, vertical/horizontal modulus of elasticity and thickness of the masonry. In this study, it was decided to proceed with a macro-level distinction of the infills in terms of shear strength. According to the results available in the literature [15], this parameter is one of the most important affecting the lateral capacity of RC frames and could also lead to local shear failure (e.g. shear failure of columns that were built without particular construction details). Five masonry infill typologies, from weak to strong, classified according to the shear capacity, were thus selected as representative of the existing masonry infill typologies used in RC residential buildings built in 1970-1980 in Italy. The results of three quasi-static cyclic tests [12-14] were used to evaluate the properties of the infill panels. The first three typologies were selected according to the classification proposed by Hak *et al.* [16]. These three typologies are referred to as infill type 1, type 2 and type 3. Infill type 1 is a weak masonry infill that was investigated in [12], in which the clay blocks and mortar were selected according to the building practice of the European earthquake-prone countries. This is a single leaf infill, constructed of horizontally hollowed brick with a 1.0cm thick plaster on each external side and thickness of 8.0cm. Infill type 2 is made up of double leaves, each constructed of horizontally hollowed bricks and 12.0cm of thickness, covered with a 1.0cm thick

external plaster and divided by a cavity of 5.0cm; infill type 3 is a single leaf constructed by vertically hollowed bricks and 30.0cm thick. The other two masonry infills, termed type 4 and 5, were selected from the experimental tests provided in [13-14], respectively. Infill type 4 [13] is an ordinary single-leaf masonry infill of 35cm thickness, made up of vertically hollowed lightweight tongue and groove clay block units, while infill type 5 [14] is a traditional single-leaf masonry infill of 15cm thickness, consisting of vertically hollowed clay block units. Considering the high variability in the selected infill typologies, it is believed that, even if some typologies that can be found in existing buildings are not represented, due for example to the variation in the thickness of the panels, the combination of five different masonry infill typologies is representative.
The geometrical variability was also considered in the definition of the infill configurations. In particular, the presence of openings in the infill panels was included. In order to provide reliable values of opening dimensions, several investigations on real buildings should be conducted. Bal *et al.*[3] investigated the characteristics of the Turkish RC building stock in the Marmara region and a mean percentage of openings in the infill panels equal to 20% was found. Furtado *et al.*[17] focused on the observation and analysis of original drawings of 80 existing buildings in Portugal and observed that the opening dimensions were between 35 and 50% of the infills' area. Similar data is not available for Italy hence the results provided by Bal *et al.*[3] and Furtado *et al.*[17] were considered. At the same time, the minimum size of the openings can be defined according to code prescriptions available in Italy [18]. In particular, the Italian code prescribes that the windowed area must not be less than 1/8 of the floor area. By considering the square meters generically assigned to each room as function of its designated use, the span length and the dimensions of the infills, the resulting opening percentage varies from 20% to 50% of the panel area. This range is consistent with the observations provided in [3,17]. As such, a percentage of openings varying between 20% and 30% of the infills' area was considered. Finally, in terms of geometrical distribution of the infills across the building height, two possible scenarios were considered: *pilotis* and uniformly infilled frames.

2.4. Simulated design approach

Table 1 summarizes the main geometrical and material parameters considered in this study to define the building portfolio. The design was carried out using a simulated design approach according to the codes in force between 1970 and 1980 [11].

Table 1 – Geometrical and material parameters of building portfolio

Description	***Variable***	***Range***
Building length X direction	L_x	[10-50] m
Building length Y direction	L_y	[8-25] m
Span length	b_x or b_y	[3-5] m
Number of stories	n	[1-6]
Interstory height	h	3.0 m
Steel yield strength	f_y	350 MPa
Longitudinal rebar diameter	ϕ_l	14 mm
Stirrup diameter	ϕ_s	8 mm
Concrete compressive strength	f_c	20 MPa
Concrete cover	c	2 mm
Column height	h_c	[300-550] mm
Beam height	h_b	[300-400] mm
Column or beam width	b_c or b_b	300mm
Masonry infill	IF	Type: [1-5]

The design process started with the analysis of continuous beams, in which both simplified equations or FEM models could be employed to determine maximum stresses on the RC beam elements. In this research study, the sum of the permanent and gravity loads is equal to 9kN/m^2 and 7kN/m^2 for typical and roof floors, respectively. The maximum axial load was calculated evaluating the area of influence for each column. The design compression stress was assumed equal to 6.5MPa, according to the prescriptions provided by the design code in force in the considered construction period. The design of the RC sections and steel reinforcement (longitudinal rebars and stirrups) was carried out using the usual equations from static analysis of RC buildings. It is worth noting that the actual design could differ from the one obtained here, as different cross sections, bars diameter, concrete cover, type of stirrups, orientation of the elements, etc., could be obtained based on the practitioners' assumptions or restrictive conditions. Since this study is not focused on the seismic risk assessment of specific single buildings, for which original drawings and visual inspections could drive towards a more reliable simulated design, design practice and expert judgment were used to finalize the design. The outcome of the simulated design approach was columns and beams heights in the range of 300-550mm and 300-400 mm, respectively. The width of both columns and beams was fixed at 300mm. The results of the simulated design are in good agreement with those provided in [3,19].

3. Numerical modeling

3.1. RC elements modeling

In the past decades, several research and experimental tests have been carried out in order to better understand the seismic response of existing RC buildings and to provide useful information for the development of advanced numerical models able to accurately simulate their seismic response. Numerical modeling of existing RC structures should be able to account for many phenomena that may occur in the structure during an earthquake, such as: material and geometrical non linearity, bar slippage, flexible joints with likely shear failure, behavior of poorly detailed and non-ductile RC frame members, premature shear failure, deficiencies in concrete core confinement due to stirrups spacing, inelasticity concentrated in the structural element ends, amongst others. The numerical models develop in this study follow the indications provided by O'Reilly and Sullivan [20] and the main features are reported in Fig. 2.

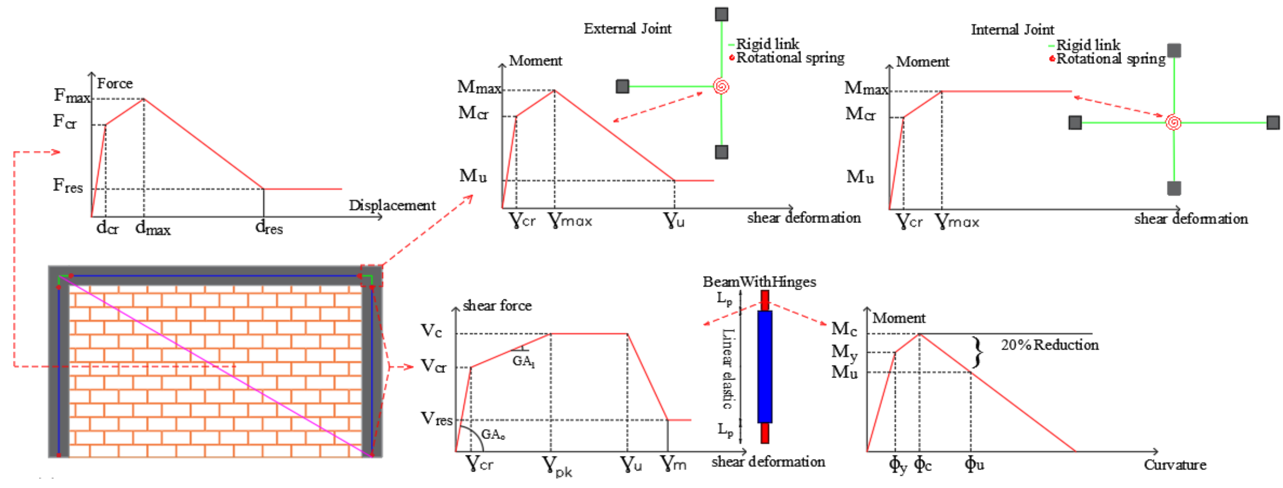


Fig. 2 Numerical modeling of RC members, beam-column joint and masonry infill

This modelling approach was chosen due to the fact that the response of each structural member, such as columns or joints, was calibrated with experimental tests available in the literature. The frame members were modelled using BeamWithHinges [21] elements, implemented in OpenSees [22]. The flexural behavior of the beam/column element is characterized by a lumped plasticity model, in which the parameters are defined according to the moment-curvature relationship proposed in [20]. The shear backbone curve is modelled according to Zimos *et al.*[23], in which a set of expressions to calculate the backbone shear deformation curve of RC element are proposed. It consists of four phases, namely initial elastic behavior, post-cracking, peak response and strength degradation. The RC joint's behavior, both for external and internal joints, was modelled by means of rigid links in the joint and a rotation spring, which represents the shear joint behavior. The hysteretic behavior and stiffness degradation were calibrated in accordance with the studies by O'Reilly and Sullivan [20] and Pampanin *et al.*[24].

3.2. Masonry infill modeling

Masonry infills were in turn modelled using a bidiagonal equivalent strut model, which consists of modelling the panel through two diagonal, compression-only nonlinear truss elements. Although more advanced numerical modelling procedures are available in the literature to account for the influence of the masonry infills, this modelling approach was preferred due to its computational efficiency and straightforwardness. However, as highlighted in Crisafulli *et al.*[25], although the global behavior of the RC infilled frames is predicted quite well, the local interaction between frame and infills is not so accurate, inducing inaccuracy in the distribution of shear and moment in the surrounding frame. Further studies should be foreseen in the future to introduce a more accurate numerical model for the infills, to better highlight the loads distribution at the corners to the surrounding frames and capture premature shear failure. Furthermore, due to the variability and uncertainties surrounding the masonry infills, the definition of the equivalent strut's hysteretic behavior was not defined according to an analytical model selected a priori, but a numerical validation was rather done for each considered masonry infill typology, using popular formulations available in the literature. In order to define the best formulations that reproduce the experimental results provided in [12-14], several equations and models proposed in literature are considered. For each of the five previously defined infill types, the best numerical model was identified and used in the numerical analysis. The parameters governing the reloading and pinching behavior of the numerical model were also defined and were found to be in accordance with those provided in [26]. The results of the numerical validation, in terms of cyclic load-displacement curves, for infill types 1 and 4 are presented in Fig. 3.

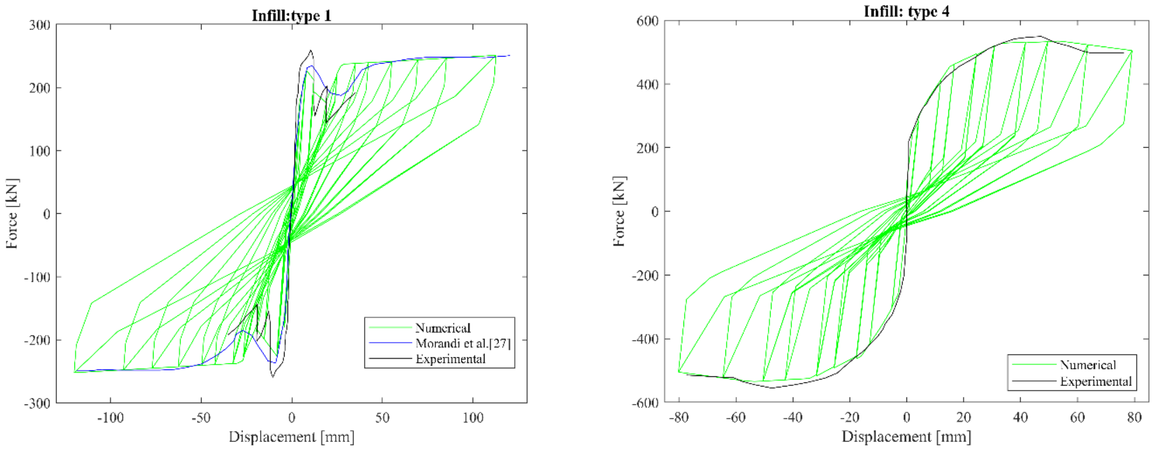


Fig. 3 Numerical modeling validation of single-story and single-bay RC infilled frame specimens

4. Pushover analysis of the portfolio frames and discussion of results

The number of bays and the length of the analyzed 2D masonry infilled RC frames vary according to the data reported in Table 1 for L_Y (8-25 m). The span length varies between 3 and 5m, with steps of 1m, while the height of the buildings varies between 3 and 18m, with steps of 3m (the interstory height). A total of 108 different geometrical configurations were defined, combining the different variables according to their ranges. All the frames were designed using the described simulated design approach and the code in force between 1970 and 1980. Five different structural configurations were considered (Fig. 4): bare frames, uniformly infilled frames (with and without openings) and *pilotis* frames (with and without openings). Moreover, as also describes previously, five different masonry infill typologies were included, leading to a total of 2088 RC frames, subdivided as follows: 108 bare frames, 900 *pilotis* frames with and without openings, 1080 uniformly infilled frames with and without openings.

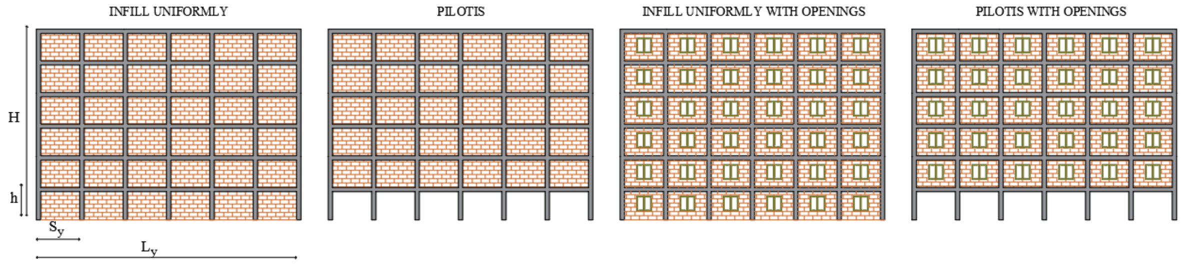
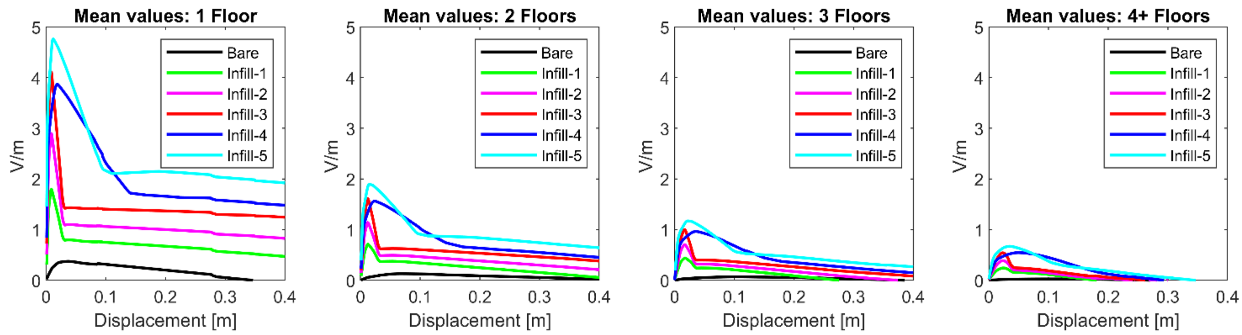
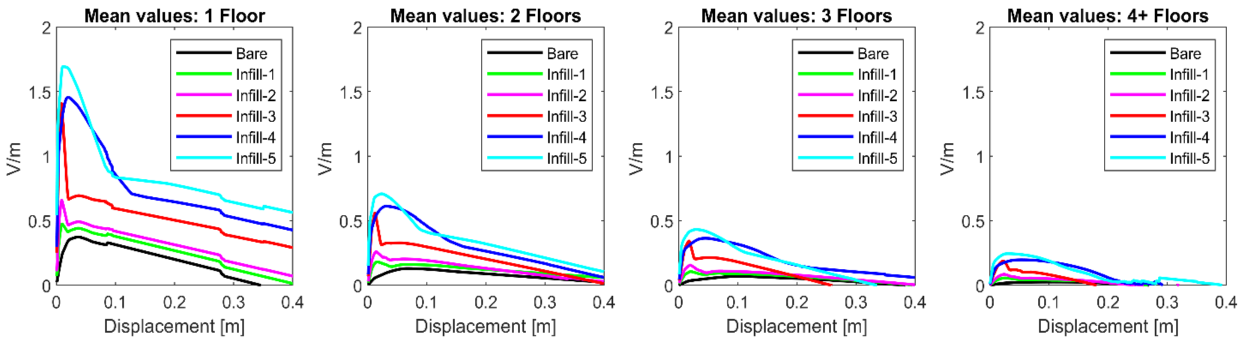


Fig. 4 Different RC infilled frame scenarios considered in the building portfolio

Pushover analysis was then carried out in each of them to evaluate their global seismic capacity. The results are presented in terms of capacity curve, i.e. seismic coefficient (base shear, V, divided by seismic mass, m) versus roof displacement, Δ. A linear distribution of horizontal forces was applied to the frames to perform the pushover analysis. In order to post-process the seismic capacity results for all of the analyzed RC frames, the results were grouped, to better observe the impact of the main considered variables in terms of mean seismic capacity and corresponding standard deviation. Specifically, the influence of the geometrical configuration, masonry infill typology and number of floors was investigated. First, the results showed that the seismic coefficients for the 4-, 5- and 6-story RC frames are comparable thus all the frames with 4, 5 and 6 stories were grouped. Then, for what concerns the structural typology, Fig. 5 reports the results for the uniformly infilled configurations (with and without openings in the infill panels) while Fig. 6 reports the results for the *pilotis* configurations (with and without openings in the infill panels). In each subplot, the normalized mean capacity curves for all the masonry infill types and the bare frame typology are presented. This representation allows to better understand, per structural typology, the impact of the infill variability on the global seismic response.



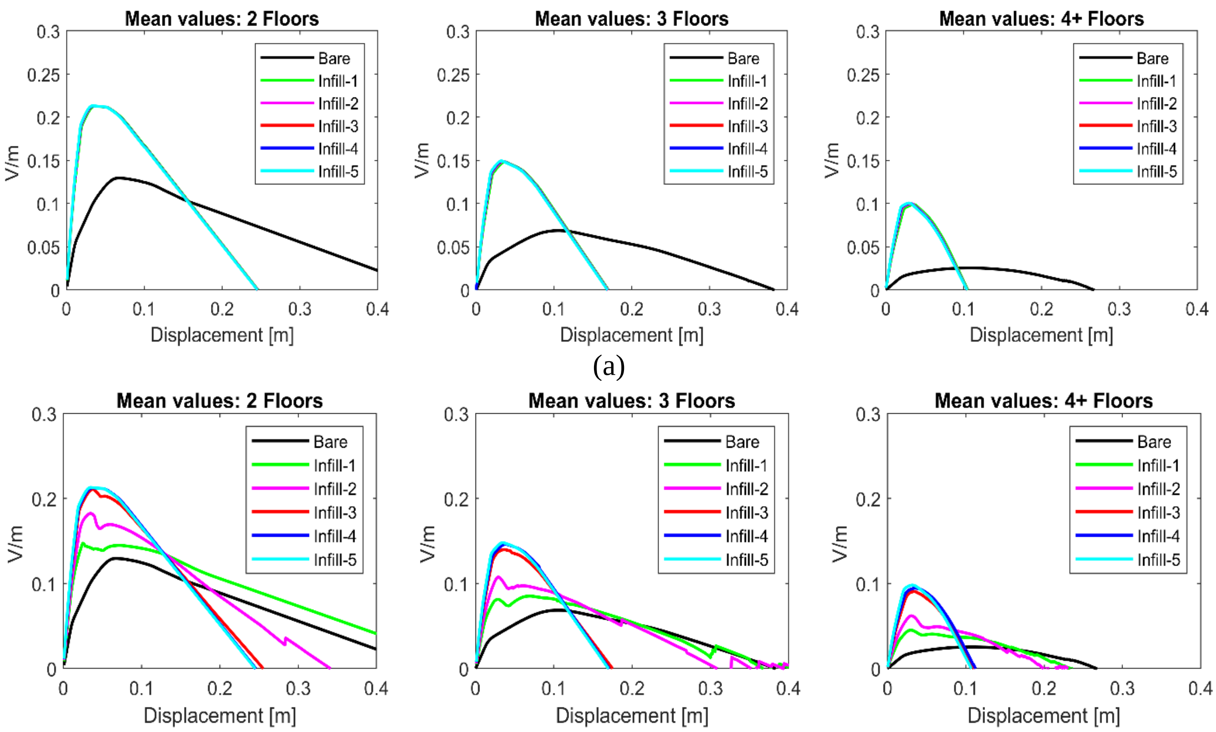
(a)



(b)

Fig. 5 Normalized mean capacity curve: uniformly infilled frames (a) without and (b) with openings

Fig. 5a shows the normalized mean capacity curve for the uniformly fully infilled RC frame typology. In terms of bare frame configuration, it was observed that as the number of floors increases from 1 to 6, the global seismic coefficient significantly reduced, going from 0.4 to 0.025. This behavior is related to the increase of the seismic mass, which is not compensated by an equal increment in the capacity of the structure, since the columns were designed only for gravity loads. The influence of masonry infills in terms of mean global seismic capacity, for all considered heights of the RC frames, is evident. The variability in the masonry infill properties significantly affects the response of the different RC frames across the building portfolio. Regardless of the number of floors, the mean maximum seismic coefficient increases of almost 63%, when comparing infill type 1 with infill type 5. Moreover, comparing frames of 1 and 4 or more floors, while keeping the same masonry typology, the maximum mean capacity decreases almost 86%. The higher capacity reduction is found when moving from 1 to 2 floors (about 61%).
Fig. 5b shows the normalized mean capacity curves for the uniformly infilled frames with openings. Given that a strength and stiffness reduction coefficient due to the presence of openings was introduced, the global seismic capacity significantly decreased, when compared to the previous structural typology. Depending on the number of floors, this reduction is in the range of 63% to 80 %, as function of the infill type. Furthermore, the mean seismic coefficients of the infilled RC frames with openings are higher than the bare ones. It is also worth noting that when considering a weak masonry infill (type 1) with openings, the global capacity is quite close to a bare frame typology, although the difference in terms of elastic stiffness is still significant. Given the number of floors, the variability in terms of global capacity due to masonry infills type is very large and, moving from infill type 1 to infill type 5, the mean seismic coefficient increases of almost 76%. Moving from 1 to 4 or more floors, the maximum mean capacity of the infilled with openings building portfolio decreases of almost 86-89%. The higher capacity reduction is found when moving from 1 to 2 floors, resulting in the range of 58-64%; the capacity reduction is slightly lower moving from 2 to 3 floors or from 3 to 4 floors, in which the capacity reduction is in the range of 39-50%, yet still significant.



(b)

Fig. 6 Normalized mean capacity curve: *pilotis* without (a) and with (b) openings typology

For what concerns the two investigated *pilotis* typologies the normalized mean global capacity curves presented in Fig. 6 show that, when there are no openings (Fig. 6a), the variability in the masonry infill typology has no effects on the global results, a consequence of the *pilotis* effect. Moreover, moving from 2 to 4 or more floors, the maximum mean capacity of the infilled buildings portfolio decreases of almost 52%, pointing out a smaller capacity reduction than the previous typologies. In particular, increasing the number of floors, one floor at a time, the capacity decreases in the range of 29-33% depending on the type of infill considered. Considering the *pilotis* typology with openings, moving from infill type 1 to infill type 5, the mean global seismic capacity increases of almost 33-60%, depending on the number of floors considered. Increasing the number of floors by a scanning step of one floor, the capacity decreases in the range of 29-50%, depending on the considered infill type. It is also worth mentioning that no relevant variations are observed between *pilotis* typologies with and without openings when the medium to strong infills (infill types 3-4-5) are considered, whereas the presence of openings seems to further reduce the maximum mean capacity when the weak and weak-to-medium infills (infill types 1-2) are considered.
Using infill type 1 as an example, Figure 7 plots all the normalized capacity curves, as a function of the geometrical configuration: the top-left subplot shows the uniformly infilled frames of different number of floors, whereas the remaining subplots represent, for each single number of floors, the normalized mean global capacity curve and the corresponding dispersion (± standard deviation). These plots provide an insight on the building-to-building associated variability, given a masonry infill type.

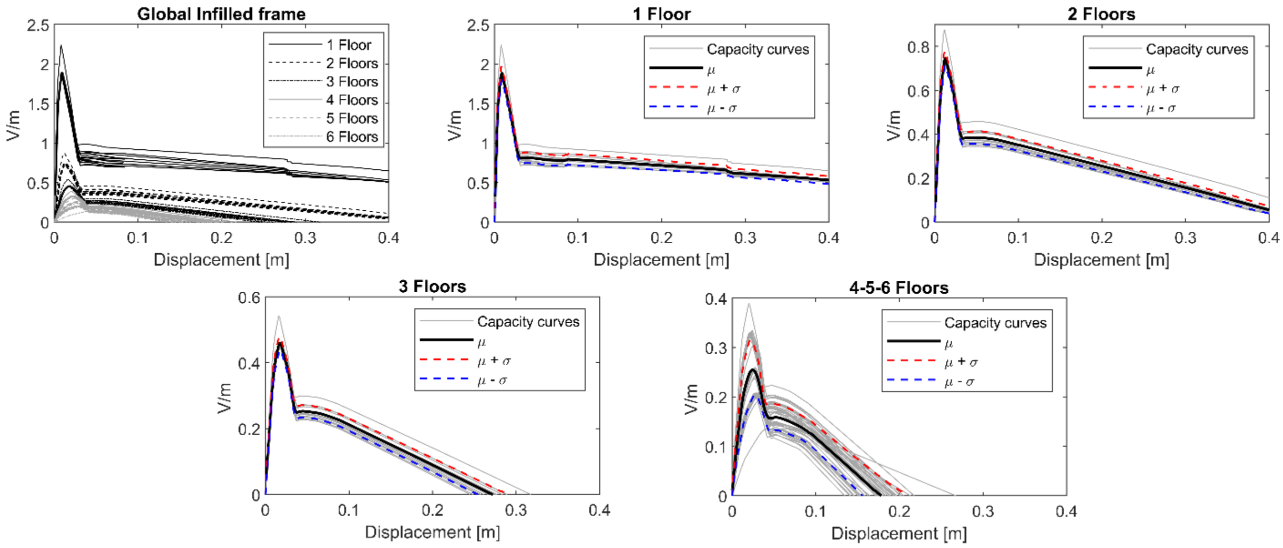


Fig. 7 Normalized mean capacity curves of infilled frames (infill type 1) of different geometrical configurations

This variability, which is, for this sample, in the range of ±7-35% with respect to the mean values, should be accounted for when performing large-scale seismic risk analyses. For the building portfolio analyzed herein, it is evident that the influence of the building-to-building variability in the seismic capacity is lower than the intra-building one, i.e. related to infill-to-infill variability. This remark confirms the relevance of considering the dispersion in the characterization of infills in seismic risk studies.

Conclusions

This study dealt with the understanding of impact of the dispersion surrounding the infill properties in large portfolios of existing infilled-RC buildings. A fully integrated Italian residential building portfolio of existing masonry infilled RC buildings was developed using statistical data and satellite images, as well as the results of several investigations and studies carried out in Italy and other Mediterranean countries. A portion of the building portfolio was then thoroughly analyzed through pushover analysis of 2D frames in order to investigate the impact of both the building-to-building and the infill-to-infill variabilities. The preliminary results reported in this study demonstrated that the variability related to the definition and modelling of the masonry infill is of paramount importance for the seismic response of different classes of buildings within a given portfolio. Although the building-to-building variability (essentially as a function of the height of the building and of the structural system) has proven relevant and should be taken into account, the infill-to-infill variability was found to have an even higher impact. As such, especially in the absence of detailed information, the uncertainty surrounding the infill characterization should be considered in large-scale seismic fragility or risk assessment endeavors and could also be essential in the process of designing a retrofitting/repairing intervention. Finally, in contrast to the geometrical variability, that might be reduced with in-situ surveys and original drawings, the infill to infill variability requires more analysis and in-situ investigations devoted to the characterization of the masonry infills that are present in the existing RC buildings.

References

[1] Bosi A., Marazzi F., Pinto A., Tsionis G. (2011): The L'Aquila (Italy) earthquake of 6 April 2009: report and analysis from a field mission. EUR - Scientific and Technical Research Reports. *Publications Office of the European Union,* DOI:10.2788/188

[2] http://dati-censimentopopolazione.istat.it/Index.aspx

[3] Bal I.E., Crowley H., Pinho R., Gulay G. (2007): Structural characteristics of Turkish building stock in Northern Marmara Region for Loss Assessment Applications. *ROSE Research Report 2007/03*, IUSS Press, Pavia, Italy.

[4] De Luca F., Morciano E., Perrone D., Aiello M.A. (2018) "MID1.0: Masonry Infilled RC Frame Experimental Database". *In: di Prisco M., Menegotto M. (eds) Proceedings of Italian Concrete Days 2016. ICD 2016. Lecture Notes in Civil Engineering, vol 10. Springer, Cham*

[5] https://www.google.com/maps

[6] Verderame G.M., Polese M., Cosenza E., Manfredi G. (2002): Vulnerability analysis of a pre-seismic code R.C. building in Catania, in seismic behavior of GLD R.C. buildings. *(E. Cosenza Ed.), CNR – GNDT, Rome, Italy*.

[7] Pecce M., Polese M., Verderame G.M. (2004): Seismic vulnerability aspects of r.c. buildings in Benevento, in The Many facets of Seismic Risk CRdC AMRA. *Eds. M. Pecce, G. Manfredi, A. Zollo. [http://www.amra.unina.it/docs.php?op=pubblicazioni]*

[8] Iervolino I., Manfredi G., Polese M., Verderame G.M., Fabbroccino G. (2006): Seismic risk for Italian typr R.C. buildings. *First European Conference on Earthquake Engineering and Seismology (a joint event of the 13th ECEE & 30th General Assembly of the ESC) Geneva, Switzerland*

[9] Verderame G.M., Ricci P., Esposito M., Sansiviero F.C. (2011) : Le caratteristiche meccaniche degli acciai impiegati nelle strutture in c.a. realizzate dal 1950 al 1980. *Atti del XXVI Convegno Nazionale AICAP "Le prospettive di sviluppo delle opere in calcestruzzo strutturale nel terzo millennio" (in Italian)*, Padua, Italy.

[10] Masi A., Digrisolo A., Santarsiero G. (2014): Concrete strength variability in Italian RC buildings: Analysis of a large database of core tests. *Appl. Mech. Mater.,* **597**, 283–290.

[11] Ministerial Decree (D.M. 30/05/1972): Design code of reinforced concrete, prestressed concrete and steel structures, *(in Italian).*

[12] Calvi G.M., Bolognini D. (2001) Seismic response of RC frames infilled with weakly reinforced masonry panels. *Journal of Earthquake Engineering,* **5**(02), 153-85.

[13] Morandi P., Hak S., Magenes G. (2018), Performance-based interpretation of in-plane cyclic tests on RC frames with strong masonry infills. *Engineering Structures,* **156**, 503–521.

[14] Cavalieri L., Di Trapani F. (2014), Cyclic response of masonry infilled RC frames: Experimental results and simplified modeling. *Soil Dynamics and Earthquake Engineering*, **65**, 224-242

[15] Perrone D., Leone M, Aiello M.A. (2017): Non-linear behaviour of masonry infilled RC frames: Influence of masonry mechanical properties. *Engineering Structures*, **150**, 875–891

[16] Hak S, Morandi P, Magenes G, Sullivan T. Damage control for clay masonry infills in the design of RC frame structures. *Journal of Earthquake Engineering*, **16**(S1), 1-35.

[17] Furtado A., Costa C., Arêde A., Rodrigues H. (2015): Geometric characterisation of Portuguese RC buildings with masonry infill walls. *European Journal of Environmental and Civil Engineering*, DOI: 10.1080/19648189.2015.1039660.

[18] Ministerial Decree (D.M. 05/07/1975): Amendments to ministerial instructions June 20, 1896, relating to the minimum height and to the main sanitation requirements of the living quarters, *(in Italian).*

[19] Silva V., Crowley H., Varum H., Pinho R., Sousa L. (2014): Investigation of the characteristics of Portuguese regular moment-frame RC buildings and development of a vulnerability model. *Bulletin of Earthquake Engineering*, **13**(5),1-36.

[20] O'Reilly G. J., Sullivan T. J. (2017): Modeling techniques for the seismic assessment of the existing Italian RC Frame Structures. *Journal of Earthquake Engineering*, DOI: 10.1080/13632469.2017.1360224 .

[21] Scott M.H., Fenves G.L. (2006): Plastic hinge integration methods for force-based beam–column elements. *ASCE Journal of Structural Engineering*, Vol. 132, No. 2, 244-252.

[22] McKenna F., Fenves G., Filippou F.C., Mazzoni S. (2000): *Open system for earthquake engineering simulation (OpenSees)*, http://opensees.berkeley.edu/wiki/index.php/Main_Page.

[23] Zimos D.K., Mergos P.E., Kappos A.J. (2015): Shear hysteresis model for reinforced concrete elements including the post-peak range. *COMPDYN 2015-5th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Crete Island, Greece.

[24] Pampanin S., Magenes G., Carr A. (2003): Modelling of shear hinge mechanism in poorly detailed RC beam column joints. *Concrete Structures in Seismic Regions*, Athens, Greece.

[25] Crisafulli F.J., Carr A.J., Park R. (2000): Analytical modelling of infilled frame structures – A general review. *Bulletin of the New Zealand Society for Earthquake Engineering*, **33**(1), 30–47.

[26] Panagiotakos M., Fardis M.N. (1996): Seismic Response of Infilled RC Frame Structures. *11th World Conference on Earthquake Engineering*, Mexico City, Mexico.

[27] Morandi P., Hak S., Magenes G. (2011): Comportamento sismico delle tamponature in laterizio in telai in C. A.: definizione dei livelli prestazionali e calibrazione di un modello numerico (in Italian). *In: XIV Convegno 'l'ingegneria sismica in Italia' (ANIDIS)*

